APPENDIX B. RED RIVER WATERWAY, LOUISIANA

B.1 Project Description

The Red River Waterway was authorized by the U.S. Congress in 1968 with the primary purpose of providing a 9-ft deep by 200-ft wide navigation channel from the Mississippi River upstream to Shreveport, Louisiana, Figure B.1. The project includes five locks and dams, with a total lift of 141 ft. Construction of the project proceeded in an upstream direction. Lock and Dam 1 was completed in the fall of 1984; Lock and Dam 2 in the fall of 1987; Lock and Dam 3 in December, 1991; Locks and Dams 4 and 5 in December, 1994. The project includes channel realignment and bank stabilization. Total project cost was approximately $1.8 billion, with about half that cost being for the five locks and dams.

Preproject river length was about 280 miles, and this was shortened about 50 miles, or 18 percent, by realignment work. Shortening has lowered flood profiles. During the May, 1990, high water on the Red River, a flood exceeding the 100-yr frequency event, peak stages at Shreveport were in the order of one to two ft below what would have occurred prior to realignment of the channel (Pinkard, 1995b).

All dam spillways have tainter gates 60-ft long for normal operation to maintain the pool during low-water periods and to pass flood flows. Lock and Dam 1 was designed to pass the design flood (the 100-yr recurrence frequency event) with one ft of swellhead, but 11 gate bays were required. Dams 2 through 5 have tainter gates and an uncontrolled ogee bay with crest set at 2 ft above upper pool level and crest lengths ranging from 150 to 315 ft. For Dams 2 through 5, economic analysis of the costs of additional gates vs the costs of fewer gates plus the costs of flowage easements for inundating additional lands indicated that it would be cheaper to use five or six tainter gate bays and an uncontrolled crest, and obtain additional flowage easements rather than to provide a larger number of tainter gates to pass the design flood with one foot of swellhead, as discussed below.

Stilling basins were designed to provide submergence of 85 percent of the conjugate depth of the entering flow and consist of a concrete slab with two rows of baffle blocks and a sloping end sill (Robertson, 1995).

All locks are 84 ft wide by 800 ft long and have sidewall port filling and emptying systems designed to limit hawser forces to less than 5 ton. Locks are sized for a design tow consisting of six barges (each 35 ft by 195 ft) and a tug. Lock lifts range from 35 ft to 24 ft, and 6-barge tows can pass through a lock in a single lockage in about 25 minutes.

On the Red River, channel velocities become too high for commercial navigation when flow is greater than the 10-year frequency flood (125,000 and 145,000 cfs at Shreveport and Alexandria, respectively). Therefore the top of lock chamber walls was set at an elevation of at least 2 ft above the 10-yr flow line (in the order of 8 ft above normal lower pool level) so that the locks are operational up to the 10-year flow frequency event. During high water, mean channel velocities are about 7 ft/sec and maximum velocities are in excess of 10 ft/sec.

Each approach at Lock 1 has a floating guide wall 685 ft long to assist tows entering and leaving the lock. When the I-wall at the lower lock approach at Lock and Dam 1 was overtopped in the 1984-85 high-water period, shortly after the project became operational, there was major sediment deposition in the vicinity of the lock. Material deposited against the lower miter gates and fell into the lock chamber when the gates were opened. Studies indicated the downstream I-wall should be raised to a higher elevation, and the wall was raised vertically using treated timbers supported by steel H beams. The timber wall extends 900 ft downstream of the miter gates and has successfully reduced the deposition that occurs in the lower approach. Some deposition still occurs, but in smaller amounts, and in areas that can be dredged more easily.

The pool at Lock and Dam 1 is at elevation 40; the dam has 11 tainter gates, and the lock is separated from the dam by an 250-ft nonoverflow section. The upstream and downstream lock approaches at Lock and Dam 1 are separated from the active flow portion of the river up to a specific stage by an earthen embankment and a concrete I-wall, Figures 10.9 and B.2.

The navigation pool at Lock and Dam 2 is at elevation 64; the dam has five tainter gates and a 190-ft uncontrolled crest at elevation 66. This structure was under construction when initial sediment problems occurred at Lock and Dam 1 in 1985, limiting modifications that could be made to Lock and Dam 2 to avoid similar problems. To separate the downstream lock approach from the main river flow, a rock dike was used the same length as the lock wall and at an elevation 10 ft above the lower pool. This configuration was designed to provide a slack-water area for the lower approach and allow some flow near the surface to enter the approach to lessen eddy action. After Lock and Dam 2 went into operation in 1987, navigation conditions in the upper lock approach proved to be difficult, as discussed in Section 9.2.

The navigation pool at Lock and Dam 3 is at elevation 95; the dam has six tainter gates and an uncontrolled weir 315 ft long with crest at elevation 97. The downstream guide wall is on the riverward side of the approach to separate the lower lock approach from the main river channel. Deposition downstream of the miter gate was still a concern, and 3-in drain pipes were installed on 3-ft centers through the lower miter gate sill to provide almost continuous flow to prevent deposition immediately downstream of the gates. This design appeared to be effective and was incorporated in Locks 4 and 5 also (Robertson, 1995).

Locks and Dams 4 and 5 have pool elevations of 120 and 145 ft, respectively. These dams have five tainter gates, a hinged crest gate 100-ft long, and an uncontrolled weir 150-ft long with crest 2 ft above normal pool level. The lower guide wall is on the river side of the approach at both locks.

B.2 Sediment

The Red River drainage basin is approximately 96,000 sq miles, and about 50,000 sq miles is above Denison Dam which traps most sediment from the upper basin. The primary source of the sediment transported on the lower Red River is from bank erosion downstream from Denison Dam (Pinkard, 1995a). The average annual suspended sediment load of the Red River is 32 million tons at Shreveport (mile 228.4) and 37 million tons at Alexandria (mile 88.6). The suspended load is roughly 25 percent fine and very fine sand and 75 percent silt. Bed load is estimated to be less than 10 percent of the total load. Bed material is predominately fine to medium sand, and the material becomes finer in a downstream direction.

Significant sediment deposition problems developed at Lock and Dam 1 in the high-water period following completion of the project in the fall of 1984: in the upstream lock approach; along the riverside lock wall; in the downstream lock approach channel; and in the lock chamber, as shown in Figure B.2.

Deposition in the upstream approach, which was a slack-water area, occurred when flows exceeded 60,000 to 70,000 cfs (the 1-yr frequency flood is 95,000 cfs), appeared to be related to the width needed for safe navigation by tows entering and leaving the upper channel entrance and also by concentration of flow in a deep natural channel along the right bank. A series of four spur dikes was constructed along the upper right bank, Figure B.3, to direct flow toward the left bank. Following construction of the dikes, maintenance dredging in the upper approach decreased significantly, from 1,024,000 cu yds in 1984-85, to 284,000 in 1985-86, and 242,000 in 1986-87. Hydrographs for the three years were comparable (Little, 1987).

Deposition in the lower approach occurred when the tailwater overtopped the downstream I-wall, resulting in eddy action in the lower approach. The I-wall is overtopped for long periods due to backwater from the Mississippi River. Material deposited was primarily very fine sands and silts, with a d_{50} of 0.07 mm. In the downstream approach, there was as much as 20 ft of deposition adjacent to the lower guide wall and 8 to 10 ft around the lower miter gate following the 1984-85 high water, Figure B.2. There was concern that deposition along the riverside lock wall would threaten stability of the wall, and sediment deposition resulted in damage to the lower miter gates. Repairs closed the river to navigation for about three months in 1985.

The elevation of the downstream I-wall (38 ft) was raised by constructing a timber wall, with top elevation of 55 extending 900 ft downstream from the miter gates. After these modifications were made, deposition downstream of the lock decreased substantially, Figure B.4. A profile showing typical deposition in the lower approach in 1985, prior to construction of the timber wall, is compared with deposition in 1987 with the timber wall in place in Figure B.4. While deposition was not completely eliminated in the lower approach by these measures, it was moved downstream to where it is not a threat to the structure and can be easily removed.

Maintenance dredging at Lock and Dam 1 is discussed in Section 10.4.

B.3 Hinged Pool Operation

Hinged pool operation can be used for sediment management as well as to reduce real estate acquisition costs. As flood levels drop, water surface slopes through a pool decrease, and sediment tends to deposit in the middle reach of some pools. Drawing the pool down at the lock and dam increases the water surface slope through the pool, providing better sediment transport. Material tending to deposit in the head end of the pool is transported farther downstream into the pool where depths available for navigation are greater.

Pool hinging to reduce maintenance dredging quantities has been tested in several pools on the Arkansas River (Corps of Engineers, 1987). Results indicated that a hinging operation has the potential to substantially reduce dredging quantities in some pools, but that to maximize benefits it is necessary to determine the optimum time to initiate and terminate dredging for each pool.

Several design factors must be considered where hinged pool operation is planned:

a. The upper gate sill must be set sufficiently low so that navigable depth is provided when the pool is lowered.

b. Velocities and cross currents in the upper lock approach may be more severe than with normal pool operation.

c. Tie-up facilities for tows along the upper approach wall must be usable at the lowered pool level.

d. Port and docking facilities, water intakes, and similar structures just upstream of the dam must be designed to avoid problems resulting from lower pool levels.

e. Rapid pool drawndown may cause bank instability.

f. Operation of the spillway gates is more complex than for normal pool operation, and this could lead to misoperation of the gates.

Locks and Dams 3, 4, and 5 on the Red River are designed for hinged pool operation, but at this time only Lock and Dam 3 is operated as a "hinged pool." A constant pool elevation of 95 ft is maintained during low flows, and as streamflow increases, the water surface at Lock and Dam 3 is drawn down to 89 ft. The water surface at the dam is maintained at this lower level until tailwater begins to control the pool level. Less land is inundated at the head end of the pool with this operation than if the pool were held at normal pool level. Comparative water surface profiles and limits of acquisition of flowage easements with and without hinged-pool operation are shown on Figure B.5.

B.4 Reaeration

Low dissolved oxygen levels below impoundments during summer low-flow periods can be very detrimental to fishery resources, and various measures are employed to alleviate the problem. At Dams 4 and 5 on the Red River, a hinged crest gate is used at one spillway bay to draw warm water from the surface of the pool and discharge it onto a baffled chute, Figure B.6. Turbulence on the chute increases the dissolved oxygen concentration.

B.5 Optimization of spillway design

Lock and Dam 1 was designed to pass the project design flood (100-yr recurrence frequency post-project flood) with one ft of swellhead. A gated dam with 11 spillway bays was needed to meet this criterion, and the widened channel cross section required in the vicinity of the lock and dam to accommodate the structures was a contributing factor to sediment deposition problems immediately after the project went into operation.

For the other four locks and dams upstream, spillway optimization studies were made to compare the cost of each additional tainter gate to costs associated with inundating additional upstream lands with swellheads in excess of one ft. Based on the optimization studies, the four upstream locks and dams were designed with fewer tainter gates than used at Lock and Dam 1, and the dams also included either an uncontrolled or hinged crest gated overflow section, or both (Pinkard, 1995b).

The procedures used in the Red River optimization studies for Lock and Dam 3 are summarized in Attachment B.1. The attachment is a copy of Appendix D to the Corps of Engineers' EM 1110-2-1605, *Hydraulic Design of Navigation Dams*, 1987.

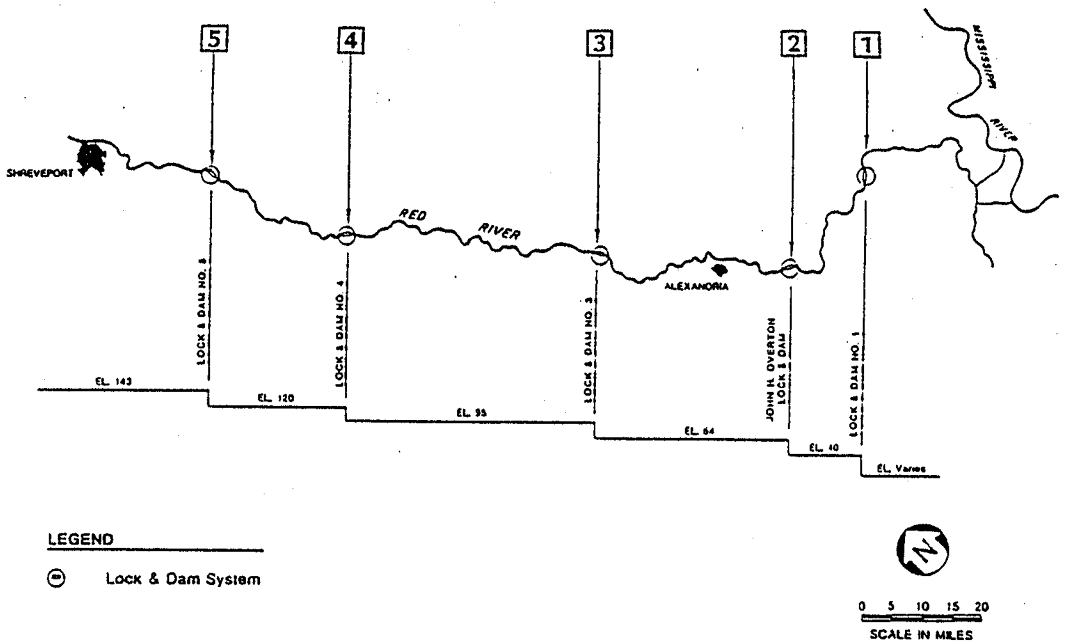


Figure B.1, Red River Waterway, Louisiana, plan and profile (Combs and Espey, 1990).

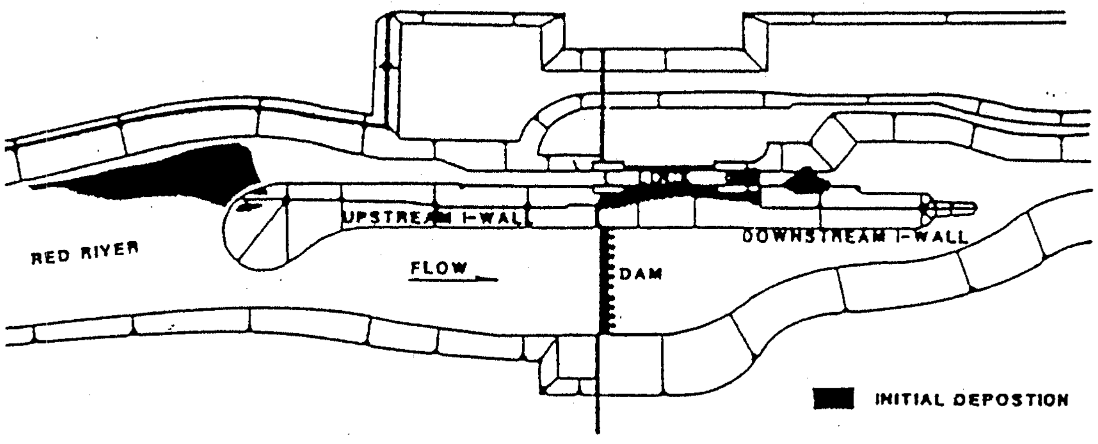


Figure B.2, Initial deposition problem areas Lock and Dam 1, Red River Waterway. (Little, 1987).

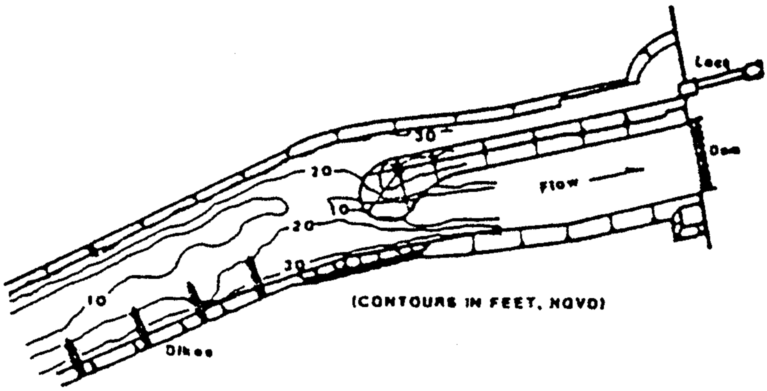


Figure B.3, Bed elevations 20 days after dike construction
Lock and Dam 1, Red River Waterway
(Little, 1987).

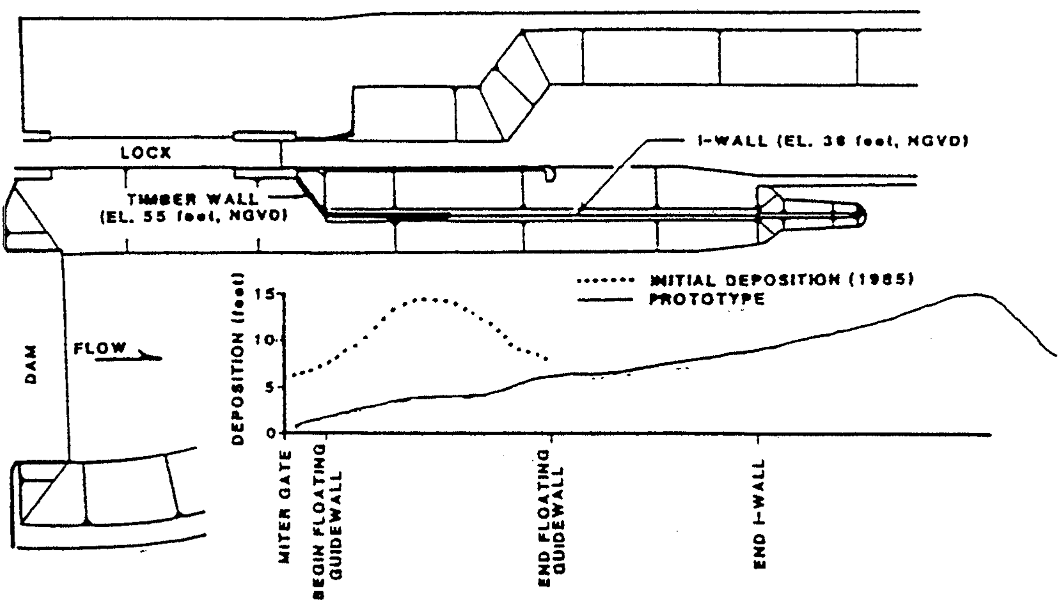
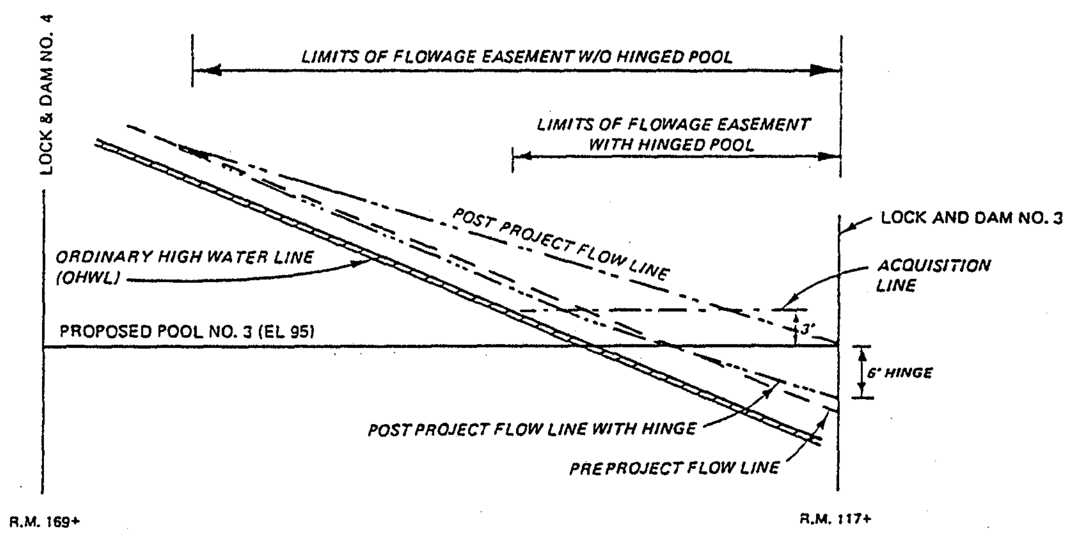
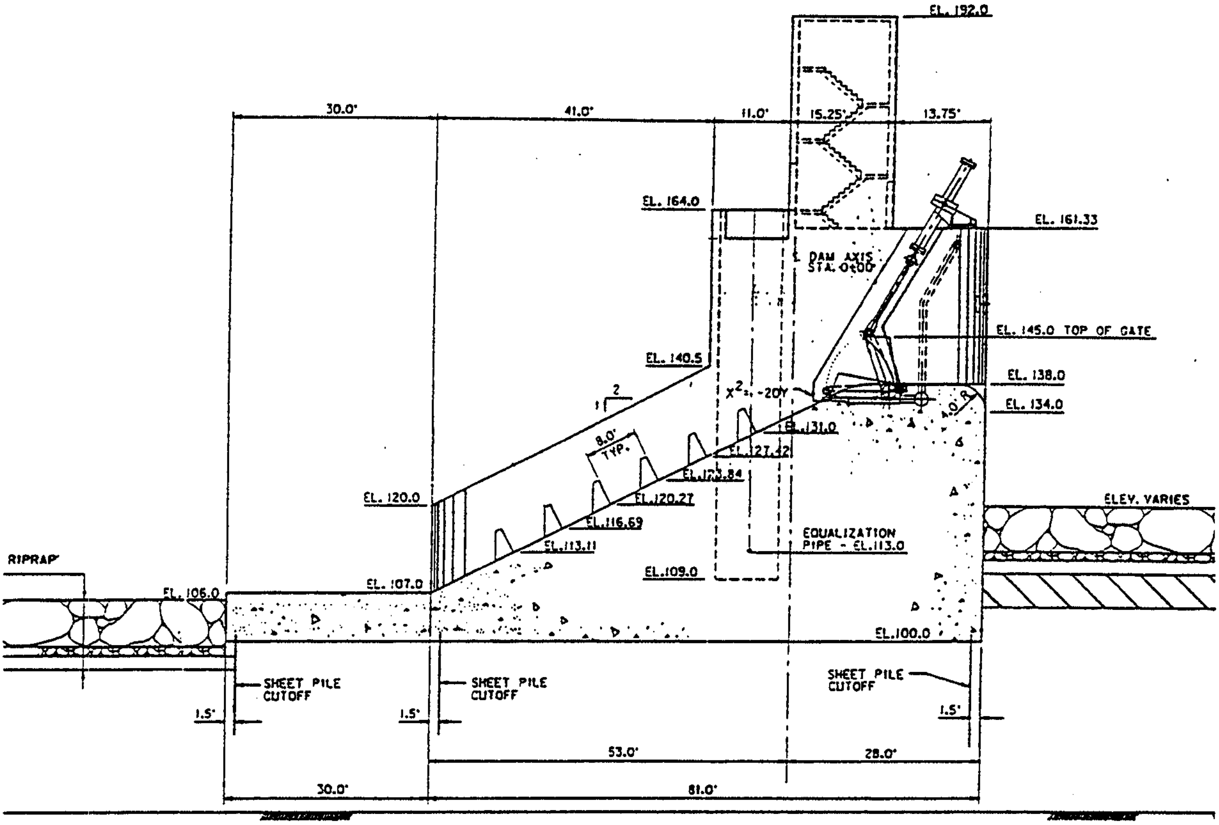


Figure B.4, Deposition below Lock and Dam 2 resulting from
hydrograph for period 19 November 1986 to 6 January 1987
(Little, 1987).

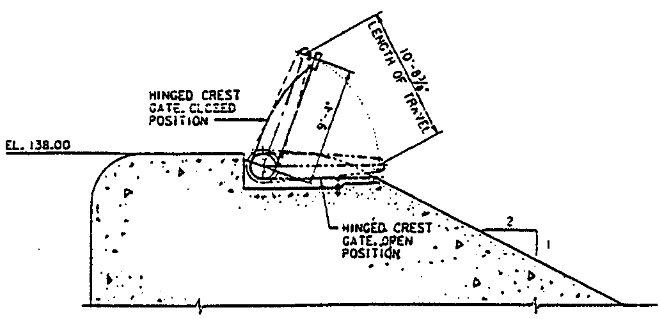


NOTE: Q = 100,000 CFS FOR THIS ILLUSTRATION

Figure B.5, Hinged pool operation, Lock and Dam 3, Red River Waterway (Corps of Engineers, 1987).



a. Section through baffled spillway chute.



b. Details of hinged gate.

Figure B.6, Hinged gate and baffled spillway chute, Locks and Dams 4 and 5, Red River Waterway (Corps of Engineers).

Attachment B.1. Typical Spillway Optimization Study, Red River, Louisiana (Appendix D, CE EM 1110-2-1605, 12 May, 1987)

1. SCOPE. This appendix summarizes the optimization studies for selection of spillway components. The goal is to select the optimum number of spillway gates and length of overflow dam. The spillway alternatives studied are tabulated in Table D-3.

2. DESIGN GUIDANCE FOR NAVIGATION DAM STRUCTURES.

a. Plans with Gates Only (No Overflow Dam). These plans provide a T-wall dam extending from last gate pier to nonoverflow embankment dam. Length of T-wall dam is governed by excavation slopes for last spillway gate bay and by location of the riverward end of the nonoverflow embankment dam. The landward end of the T-wall dam must be embedded in the riverward end of the nonoverflow embankment dam. The tops of abutments and T-wall dams must be above the headwater for the project design flood plus wave runup. Provide minimum training wall downstream of last gate bay.

b. Overflow Dam Plans with Weir 300-, 600-, and 1,200-foot Crest Lengths. These plans provide concrete overflow dam from the last gate pier to the overflow embankment dam. Length of concrete overflow dam is governed by excavation slopes for last spillway gate bay and by the riverward end of the overflow embankment dam. The overflow embankment dam was extended landward so that total length of concrete overflow plus embankment overflow is 300, 600, 1,200 feet, or other selected lengths. Easy vertical transition from overflow embankment to nonoverflow embankment has been provided. For some instances with four, five, and six gate bays, stone will not resist the overflow velocities on the downstream edge of the embankment crown, and a concrete section must be provided. Minimum training wall downstream of last gate bay must be provided.

c. Spillway Gate Piers. The trunnion anchorage elevation can be the same for all gate arrangements since it is related to tailwater.

d. Riprap. Riprap that is needed for each dam arrangement must be provided. A complete layout plan for each dam arrangement must be developed.

e. Top of Lock Walls. The top of lock walls will be eight feet above the normal upper pool for all gate arrangements. This elevation will provide substantially more than two-foot clearance above the headwater for a 10-year flood for all gate arrangements.

f. Stilling Basins and Gated Weirs. The stilling basin will have the same dimensions in an upstream-downstream direction regardless of the number of gates. The gated crests will also have the same dimensions regardless of the number of bays.

3. FLOWAGE EASEMENTS.

a. Some of the spillways would raise flood heights above preproject elevations. Assume that flowage easements are required on all lands above the ordinary high-water line on which flood heights are increased.

b. The channel realignments on this waterway would reduce the overall river length from the mouth of the Black River (1967 mile 34.2) to Shreveport (1967 mile 278) by 48 miles. This shortening will cause a reduction in flood elevations, and the reduction at the Lock and Dam 3 site is estimated to be 2.2 feet. This postproject reduction of 2.2 feet was taken into account when determining whether a given spillway arrangement would raise postproject flood levels above preproject levels. For example, the six-gate, 315-foot-weir spillway would cause a headwater elevation 2.2 feet above postproject tailwater elevation for the project design flood (PDF). However, this spillway would not raise flood heights since the postproject tailwater elevation is estimated to be 2.2 feet below the preproject tailwater elevation.

c. Table D-2 shows how much various spillway arrangements would raise the PDF (248,600 cfs) above preproject level at the damsite and the land acreages on which the PDF would be raised. The calculations showed that the following spillway arrangements would not raise the PDF above preproject conditions.

Number of Gates	Length of Overflow Dam, feet
4	1,510 and longer
5	935 and longer
6	315 and longer
7	0 and longer
8	0 and longer

d. It is proposed to acquire flowage easements up to elevation 98, which is three feet above the navigation pool elevation and one foot above the top of the overflow dam. When a postproject discharge reaches this headwater elevation at the damsite, the water-surface profile upstream will be higher than the flowage easement elevation 98 throughout Pool 3. The postproject discharge will be 178,000 cfs when the headwater elevation at the damsite is 98, and this discharge has an average recurrence interval of about 33 years.

e. The preproject profile for 178,000 cfs was calculated and compared with the postproject profiles for this discharge for the various spillway arrangements. The postproject profiles for the six-, seven-, and eight-bay spillways were equivalent to or lower than the preproject profile. Since the 178,000-cfs discharge would be only about a foot above the top of the overflow dam, the length of overflow dam does not have a significant effect on the headwater elevation. Table D-1 shows how much various spillway arrangements would raise the 178,000-cfs discharge above preproject level at the damsite and the land acreages on which this discharge would be raised.

4. LEVEE RAISING. The following spillway arrangements would raise the PDF by a foot or more above preproject and would require raising the flood-control levees adjacent to Pool 3 to provide the preproject level of protection.

Number of Bays	Length of Overflow Dam, feet
4	None
4	300
4	600
4	1,200
5	None
5	300
5	600
6	None

The entire length of this levee would be raised by the amount of height that the postproject PDF is raised above preproject at the mouth of Saline Bayou. The levees would be raised to the same height above the postproject PDF as they were above the preproject PDF.

5. COMPARATIVE COSTS. Detailed cost estimates were calculated for each of the alternative spillway arrangements using October 1982 price levels. These estimates are summarized in Table D-3.

6. CONCLUSIONS AND RECOMMENDATIONS.

a. The alternative consisting of a six-bay spillway and 315-foot overflow dam is the least costly considering all costs and is the selected spillway. The lock and dam structure costs for some of the alternatives were less than for the selected plan, but their costs for additional flowage easements and levee raising caused their total costs to be higher.

b. The recommendations for this site-specific study is to proceed with the alternative consisting of six-bay spillway and 315-foot overflow dam design.

TABLE D-1

Spillway Arrangements That Would Raise 178,000 cfs Above Preproject

Spillway Arrangement: No. of Bays	Spillway Arrangement: Length of Overflow Dam, feet	Height of Post-project 178,000 cfs above Pre-project 178,000 cfs at Damsite feet	Flowage Easements Required on Main Stem acres	Flowage Easements Required on Tributaries Approx. acres
4	All	2.0	7,000	6,910
5	All	0.9	7,000	6,910

TABLE D-2

Spillway Arrangements That Would Raise the PDF Above Preproject

Spillway Arrangement		Height of Postproject PDF above Preproject PDF at Damsite, feet	Flowage Easements Required on Main Stem acres	Flowage Easements Required on Tributaries Approx acres
No. of Bays	Length of Overflow Dam, feet			
4	None	5.3	8,500	6,910
4	300	2.8	8,241	6,910
4	600	2.0	8,147	6,910
4	1,200	0.6	7,000	6,910
5	None	2.4	8,273	6,910
5	300	1.2	7,000	6,910
5	600	0.7	7,000	6,910
6	None	1.0	3,328	3,075
6	300	0.2	—	—

TABLE D-3

Comparative Costs

Spillway Alternative		Lock and Dam Structure Costs	Additional Flowage Easement Costs	Levee Raising Cost	Total Comparative Cost
No. of Bays	Length of Overflow Dam, feet	In Dollars Rounded to Nearest Tenth of a Million			
4	0	157.6	11.6	24.7	193.9
4	300	154.8	11.4	12.1	178.3
4	600	156.5	11.3	8.0	175.8
4	1,200	158.1	10.4	Min	168.5
4	1,510*	158.9	10.4	Min	169.3
5	0	163.8	11.4	10.8	186.0
5	300	162.0	10.4	4.9	177.3
5	600	162.4	10.4	Min	172.8
5	935**	163.3	10.4	0	173.7
5	1,200	164.5	10.4	0	174.9
6	0	170.0	4.8	3.4	178.2
6	300	168.0	0	0	168.0
6	315†	168.0	0	0	168.0
6	600	168.6	0	0	168.6
6	1,200	170.7	0	0	170.7
7	0	176.3	0	0	176.3
7	300	174.3	0	0	174.3
7	600	175.9	0	0	175.9
7	1,200	179.3	0	0	179.3
8	0	183.8	0	0	183.8
8	300	182.3	0	0	182.3
8	600	183.8	0	0	183.8
8	1,200	187.6	0	0	187.6

* Structure costs were extrapolated. This alternative would not raise the PDF.

** Structure costs were interpolated. This alternative would not raise the PDF.

† This is the selected alternative. It would not raise the PDF. The six-bay spillway and 315-foot overflow dam was selected over the six-bay spillway and 300-foot overflow dam because the latter alternative would raise flood heights slightly above preproject conditions. No additional costs were shown in the table for additional flowage easements and levee raising for this slight rise in flood heights because they would be of questionable accuracy. However, the 315-foot overflow dam has the advantage of not raising flood heights, while the 300-foot overflow dam could be difficult to defend since it will raise flood heights to some extent.